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GROUP EFFECT ON UPLIFT CAPACITY OF DRILLED PIERS IN SAND

Report No 86-276-K

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ABSTRACT

Narrow based lattice type transmission towers require constructing closely spaced footings which could experience reduction in uplift capacity due to group effect. This report summarizes the results of uplift tests on 3 pairs of drilled footings spaced 1 and 1-1/2 diameters apart. Analysis of the test data showed a reduction in uplift coefficient Ku value by 30 to 35 per cent due to the overlapping zones of influence for the two footings. Additional tests of near full size footings in sand and clay are needed to support the findings of this study.

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GROUP EFFECT ON UPLIFT CAPACITY OF DRILLED PIERS IN SAND

INTRODUCTION

Narrow based towers are being considered for use in areas of restricted right-of-ways and to increase the efficiency of land use in urban centres.

Single steel poles have been used in areas where the improved appearance is needed in addition to the efficient use of right-of-way. The cost of these steel poles are prohibitive for their use in all areas. Therefore a new design of narrow based lattice towers are being considered. Typically the base of these towers is about 3 m square. The foundations for this type of narrow based tower are required to resist large loads caused by wind and ice on conductors and structure. Two possible schemes of foundation for the narrow based towers are (i) four single footings, one for each tower leg and (ii) one single foundation for the tower.

Under the first category, four cylindrical auger footings, 1.5 m in diameter (spaced about 3.0 m centre to centre) are proposed. The wind loading on a tangent tower puts 2 footings in compression and 2 footings in uplift, which could reverse with the direction of wind. In the case of angle tower or turning tower, footings will be under sustained loading of compression or uplift depending on the orientation of tower legs to the resultant forces.

The possibility of a group effect for footings in uplift has previously been suggested/2/ but the data from field load tests are very limited. The study of augered footing pairs in sand subjected to uplift forces has not received much attention in the literature.

At the request of Mr. A.R. Collins of Stations and Transmission Design, the Civil Research Department carried out field load tests on closely spaced drilled piers in sand at Essa TS, to assess the group effect on uplift capacity as related to narrow based tower foundation design.

FOUNDATION CONSTRUCTION

The test program consisted of installing and testing 3 pairs of drilled piers in the vicinity of an earlier test location just outside Essa TS (see Figure 1). Results of soil tests performed at this site as a part of the previous uplift test program are used for analyzing the test data obtained in this test program.

The test footings (0.7 m diameter by 3.7 m deep) were constructed by a boom truck equipped with a Mark IV digger head and continuous flight auger (Figure 2). A 3.6 m 1000 MPa number 14 (45 mm) Dywidag bar with a square plate threaded onto the bottom was cast in the centre of each foundation as a pulling rod (Figure 3). Type 30 (high early strength) concrete was specified so that the first footing of each pair cast would harden sufficiently that after 1 hour the other footing of the pair could be augered without the fear of weakening the set concrete.

A super plasticizer was used to produce a 200 mm slump so that the concrete would flow easily with a minimum amount of vibration (Figure 4). The spacing and dimensions of the footings (as constructed) are as shown in Figure 5 and Table I respectively. The footings were cylindrical and with vertical walls and were cast to the surface except as noted on Table I.

Telltale type heave gauges were installed around the footing in the locations shown in Figure 5. Each telltale consisted of a steel rod with a wooden disk fastened at the bottom and was installed in a 100 mm augered hole of either 0.6 m or 1.8 m depth. A 50 mm diameter PVC pipe was installed over the steel rod and terminated at about 100 mm above the wooden plate so that the soil movement at the installed depth could be measured. The void space in the hole outside the PVC pipe was backfilled with the augered material and rodded in several lifts.

SOIL PROPERTIES

The results of the dynamic cone tests which are summarized in Table III, indicate that the soil conditions were uniform over the test area. Dynamic cone tests may be equated to Standard Penetration Tests (SPT) by dividing the blow count by 1.5/1/. The soil density over the test depths borders between compact and dense. In the earlier work at this site/1/, the nuclear density tests at the surface and at depths of 0.6 m gave an average density of 1680 kg/m^3. Shear box tests performed on the sand/1/ at this density yielded a friction angle of 36°. The augered holes remained dry indicating the water table to be below 3.7 m. Earlier work/1/ had reported the water table to be at 6.1 m.

TESTING

All 3 pairs of footings were tested in uplift, one pair at a time. Footings 1, 2 and 3 were paired with 4, 5 and 6 respectively.

Two 9 m long beams were positioned side by side over the footing pair leaving a clear space of 3 m or more from the edge of the footing to the edge of the timber support pad. The pulling bar embedded in each test

footing was extended by a 2 m section of #14 Dywidag rod. The uplift force was applied to each footing of the pair by two hydraulic jacks stacked to allow a total of 150 mm travel. The footing pairs were uplifted simultaneously by applying the same magnitude of load for each footing. The applied uplift force was measured by a calibrated load cell (Figure 6). The LVDT and the dial gauges fastened to a 6 m long batter board were used for measuring the footing and telltale movements (Figure 7). The LVDT and load cell output was fed to a pair of X-Y plotters (Figure 8), which instantaneously produced a load deflection curve for each footing.

TEST RESULTS AND ANALYSIS

The uplift load and the measured deflection at failure for each footing are summarized in Table II. The load-deflection curves for each footing pair are shown in Figures 9 to 11. The measured soil heave around the footings as determined by telltale movement is shown plotted against the applied uplift load in Figures 12 to 14. The soil movement around each footing pair is plotted against the deflection of each footing in Figures 15 to 20. Telltale and footing deflection at ultimate load and 65-70 per cent of ultimate load are shown in Figure 21.

The following observations can be made from the above data:

1. Load-Deflection Behaviour: Most of the test footings failed suddenly with deflections of only 5 mm to 6 mm, most of which occurred after the applied load reached 65% of the ultimate load. This is less than the deflection at failure (12 mm to 25 mm)recorded in earlier tests on single footings/1/. Footing 5 experienced more creep than the other footings as shown in Tables IV to VI. Footing 2 did not fail at the same load as Footing 5. It was reloaded individually to failure.

2. Soil Heave: At all loads the soil heave measured by the telltales is less than the footing deflections (Figures 15 to 20), indicating that there was slippage between the footings and the surrounding soil.

   At near failure , the difference between measured footing heave and measured soil heave is less for the more closely spaced footing pair (Figure 15 and 16) than for the other footing pairs (Figures 17 to 20). This suggests that the more closely spaced footing may be behaving as a composite footing in conjunction with the soil in between them.

   Near the failure load, the soil heave is greater between the footings than the soil heave measured outside the footing pair as shown in Figures 17 and 18, which suggests that there is some overlap between the zones of influence from each footing in the pair.

UPLIFT CAPACITY

The uplift capacity of a footing is partly derived by its weight and partly by the soil resistance. Uplift capacity of a cylindrical drilled pier is estimated by using Ku theory/2/ as follows:

$$Q_u = W + 1/2\,\pi BD^2\,\gamma\ Ku \tan \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where Q_u = Uplift capacity

W = Weight of footing

B = Diameter of footing

D = Depth of footing

γ = Effective unit weight of soil

ϕ = Angle of internal friction of soil

and Ku = Empirical uplift coefficient which depends on D/B ratio and friction angle.

Uplift capacity of footings that are closely spaced together and loaded simultaneously can experience a reduction in their capacity due to the overlapping of the influence zones. This reduction will be only in their soil resistance component and not in their weight component.

Assuming that the soil failure mode for each footing in a group test is similar to that for an individually loaded footing of the same size, any reduction in the soil resistance can be expressed in terms of reduction in Ku with all the other parameters in Equation (1) remaining constant.

Table VII shows the Ku values back-calculated from the results of uplift tests on single footings spaced far apart/1/. Only 3 test footings with D/B ratio similar to the new test footings constructed for this study are considered for comparison purposes. An average value for Ku of 2.83 is obtained.

Table VIII summarizes the measured failure loads, estimated uplift capacity using Ku of 2.83 and the back-calculated Ku value from the measured uplift loads. Table IX and Figure 22 show the group effect on Ku values. A reduction in the Ku value of 35 per cent and 30 per cent were obtained, for footings clear spaced by 1 diameter and 1-1/2 diameter respectively. Ku values for Footings 2 and 5 (spaced 1-1/2 diameter) were not included in this comparison because the two footings did not fail together.

For the very closed footings,a composite failure surface may govern the uplift capacity of the group, in which case the soil in between the footings will act as a part of one single large foundation. The group uplift capacity can be estimated by treating it as a single footing and using a Ku value corresponding to the D/B ratio of the group. Figure 24 shows how the Ku value is back-calculated for footing pairs assuming a composite shear failure surface. The back-calculated Ku value for one diameter spacing is 1.62 as compared to 1.85 back-calculated from the test results (Table IX). The Ku value of 1.42 back-calculated for 1-1/2 diameter spaced footings is much smaller than the Ku value of 2.0 back-calculated from the test results (Table IX). These calculations suggest that while a composite failure mode as a footing group is likely for one

diameter spaced footings, it is unlikely for the footings spaced at 1-1/2 diameter or more. The observations of ground heave at the failure load (Figures 21 and 23) support this conclusion.

CONCLUSIONS

The uplift capacity of a pair of cylindrical drilled piers in sand, when loaded simultaneously can be equal to or smaller than the sum of their individual capacities depending upon the clear spacing between the footings. For closely spaced footings, the reduction in capacity can be expressed as a reduction in Ku value used in uplift capacity calculations.

From the results of field load tests at the Essa site, with footing pairs of 0.6 m diameter by 3.6 m depth, a reduction in Ku value by 35 per cent and 30 per cent are estimated for footing spacings of 1 diameter and 1-1/2 diameter respectively.

The soil heave measurements at depth suggest that footings spaced at 1 diameter may be behaving as a composite footing with soil in between them uplifted.

Actual reduction in uplift capacity of closely spaced footings for a narrow based tower will depend upon the size of foundations, D/B ratio and the clear spacing. Near full-scale footings should be tested to confirm the extent of reduction in uplift capacity estimated on the basis of the findings of this study as the soils at this test site were competent and test footings were above the ground water table with no caving encountered.

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1. Cragg, C.B.H. "Field Uplift Tests in Cohesionless Soils". Research Division Report No 85-186-K, November, 1985.

2. Meyerhof, G.G. and Adams, J.I. "The Ultimate Uplift Capacity of Foundations". Canadian Geotechnical Journal, Vol 5, No 4, 1968.

TABLE I

DIMENSIONS OF TEST FOOTINGS AS INSTALLED

Footing No	Footing Depth (m)	Diameter (m)	Remarks
1	3.66	0.74	
2	3.66	0.69	
3	3.66	0.71	
4	3.66	0.71	
5	3.68	0.71	
6	3.71 (3.51)	0.74	Footing slightly belled Concrete surface 0.20 m below ground surface

TABLE II

SUMMARY OF UPLIFT TEST RESULTS

Footing No	Failure Load (kN)	Weight of Footing (kN)	Footing Uplift at Failure (mm)	Remarks
1	355	36	5.7	
2	580	31		Uplift not measured at failure
3	400	33	6.5	
4	400	33	6.5	
5	370	33	8.0	Failure during hold for creep
6	380	34	5.0	

Failure load measured by load cell
Footing uplift measured by LVDT

TABLE III

TWO INCH CONE PENETRATION DATA

Depth	Hole Number 1 N_{Cone}	2 N_{Cone}	3 N_{Cone}	4 N_{Cone}	5 N_{Cone}	6 N_{Cone}
0						
1	5	4	4	4	3	3
2	10	5	5	9	5	5
3	20	20	17	25	18	23
4	27	27	25	27	30	28
5	30	27	28	32	34	40
6	39	30	31	43	34	40
7	42	38	34	38	39	31
8	41	33	35	40	42	42
9	39	36	31	42	37	52
10	39	36	35	52	38	42
11	36	36	37	40	34	42
12	27	31	35	53	29	43
13	18	25	31	41	23	40
N_{Cone} Average over Footing Depth	29	27	27	30	29	34

N_{Cone} averaged for all footings is 29

N_{Cone} averaged for all footings from earlier work/2/ is 36

TABLE IV

DEFLECTION OF FOOTING SET 1 AND 4 DURING LOADING

Time in Minutes from Start	Applied Uplift Force per Footing in Kn	Deflection in mm Measured by Dial Gauges	
		Footing No 1	Footing No 4
0	0	0	0
2	70	0.08	0.08
4	90	0.18	0.15
7	135	0.38	0.48
9	180	0.64	0.91
10	225	1.17	1.32
11	265	1.83	1.96
12	310	2.59	2.62
14	355	4.57	5.08
16	355	5.65	5.69
	400	uplift failure	uplift failure

TABLE V

DEFLECTION OF FOOTING SET 2 AND 5 DURING LOADING

Time in Minutes from Start	Applied Uplift Force per Footing in kN	Deflection in mm Measured by Dial Gauges	
		Footing No 2	Footing No 5
0	0	0	0
1	45	0.05	0.10
3	90	0.23	0
4	135	0.43	0.25
5	180	0.79	0.53
6	225	0.90	1.02
7	265	1.55	1.50
9	310	1.85	1.93
10	310	2.36	2.36
12	310	2.54	2.72
14	355	3.63	4.85
19	355	5.26	6.40
21	380	6.43	9.37
26	380	9.42	uplift failure
	0		No 5 was disconnected
	580	uplift failure	

TABLE VI

DEFLECTION OF FOOTING SET 3 AND 6 DURING LOADING

Time in Minutes from Start	Applied Uplift Force per Footing in kN	Deflection in mm Measured by Dial Gauges	
		Footing No 3	Footing No 6
0	0	0	0
0.5	45	0.36	0.13
1	90	-0.38	0.10
2	135	0.33	0.38
3	180	0.03	0.71
4	225	0.23	1.07
5	265	1.14	1.78
6	310	1.80	2.36
7	310	1.96	2.67
10	310	2.08	2.77
11	355	4.01	3.86
16	355	5.18	4.88
18	355	5.36	5.00
	380	uplift failure	uplift failure

TABLE VII

Ku VALUES BACKCALCULATED FROM UPLIFT TESTS
ON SINGLE FOOTINGS/1/ WITH D/B RATIO 4.7 TO 5.0

Load Test Number	Depth (m)	Diameter (m)	D/B Ratio	Failure Load (kN)	Backcalculated Ku
7	3.28	0.61	5.0	409	3.1
8	3.12	0.61	4.7	307	2.6
15	2.54	0.51	5.0	186	2.8

Average $Ku_{single} = 2.83 \pm 0.25$

TABLE VIII

EXPECTED FAILURE LOAD ASSUMING NO GROUP EFFECT

Footing No	Depth (m)	Diameter (m)	Failure Load Measured (kN)	Expected Failure Load* (kN)	Back-calculated Ku**
1	3.66	0.74	355	574	1.7
2	3.66	0.69	580***	533	3.1***
3	3.66	0.71	400	549	2.0
4	3.66	0.71	400	549	2.0
5	3.68	0.71	370	555	1.8
6	3.51	0.74	380	529	2.0

* Using Ku of 2.83, from the results of earlier tests, see Table VII

** From the results of uplift test on footing pairs in this program.

*** Footing 2 did not fail at 370 kN when it was loaded simultaneously with Footing 5. Then it was reloaded to failure as a single footing.

TABLE IX

GROUP EFFECT ON Ku VALUES

Footing No	Clear Spacing (m)	Clear Spacing/Avg Diameter	Avg Ku_{group}	Avg Ku_{group}/ Avg Ku_{single}	Reduction in Ku Due to Group Effect
1,4	0.65	0.90	1.85	0.65 ± 0.05	35 ± 1.7%
3,6	1.03	1.45	2.0	0.71 ± 0.0	29%

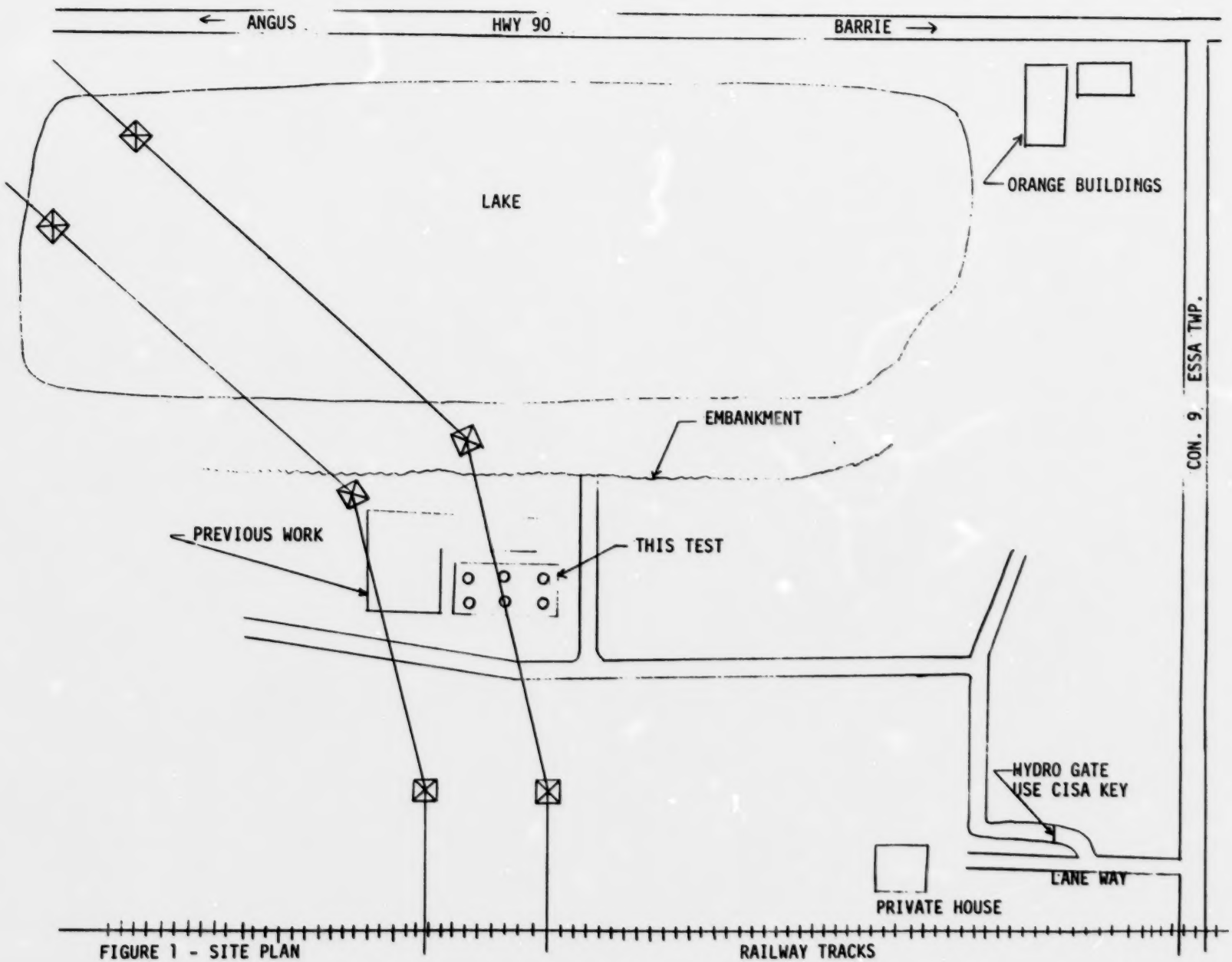

FIGURE 1 - SITE PLAN

172465 RC

FIGURE 2 - Auger (0.6 m Diameter) Used to Auger Holes for the Footings

FIGURE 3 - #14 Dywidag Bar Being Centred in the Footing Hole

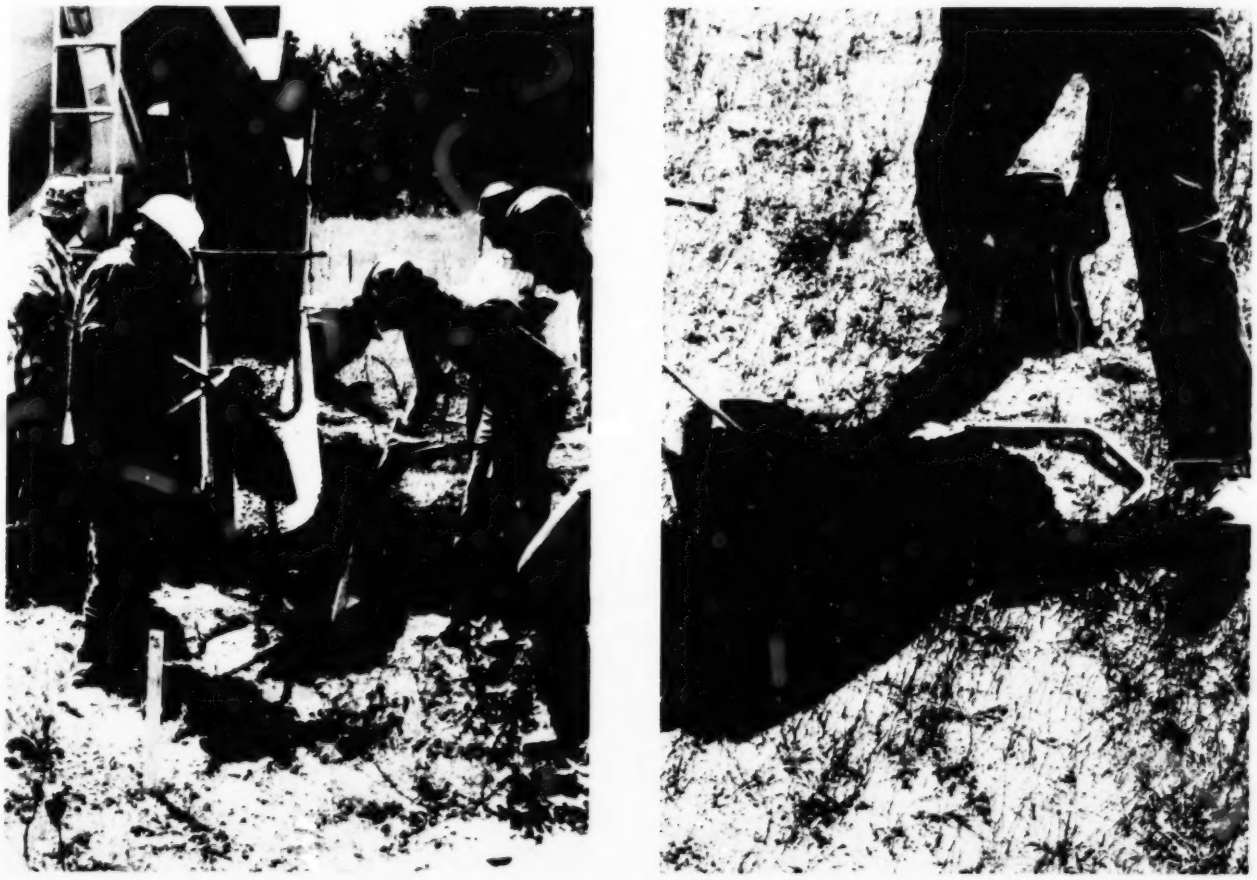

FIGURE 4 - High Slump 30 MPa Type 30 Concrete
was used for the Footings

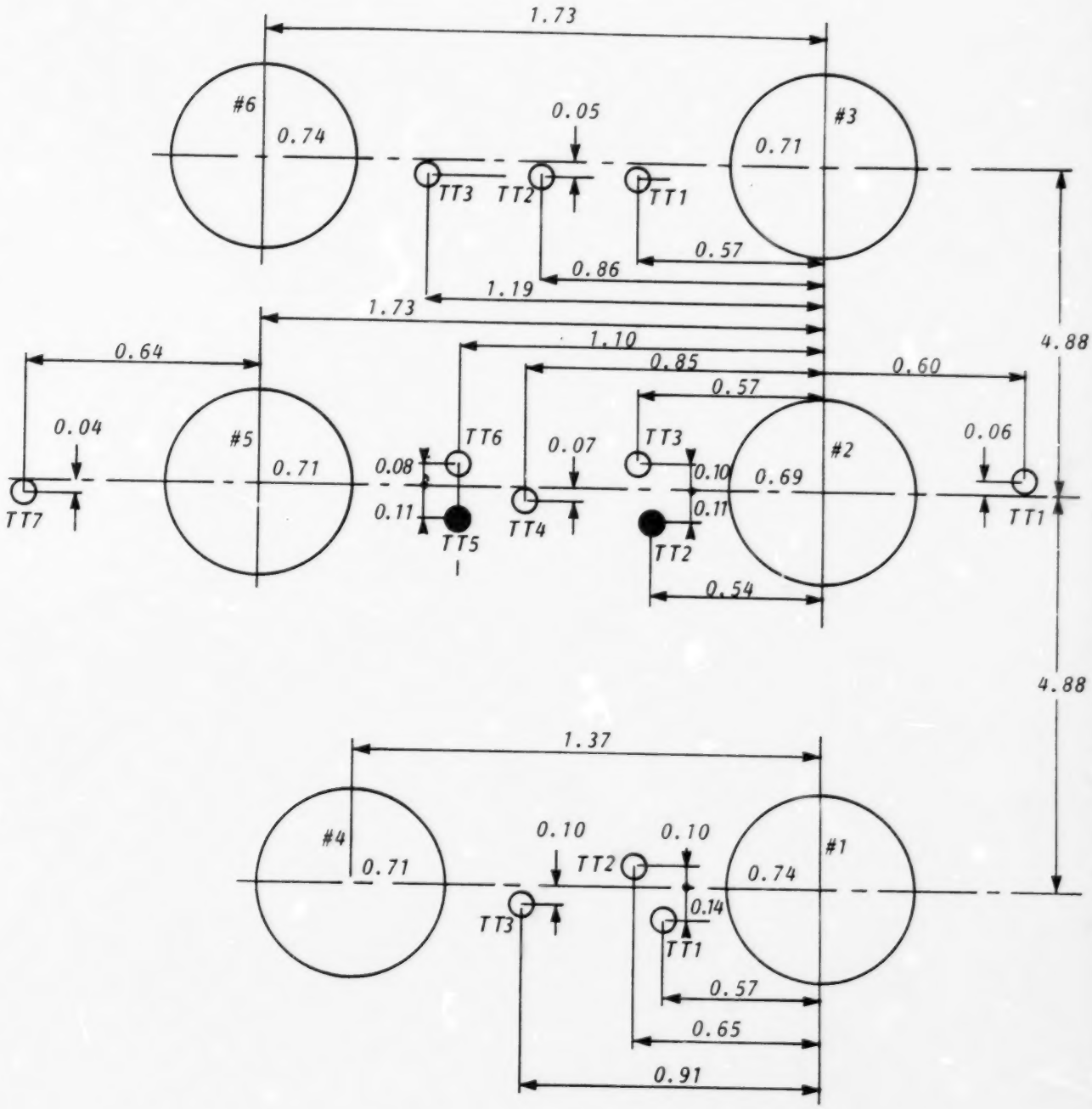

FIGURE 5

PLAN VIEW OF FOOTINGS AND TELL-TALES AS CONSTRUCTED

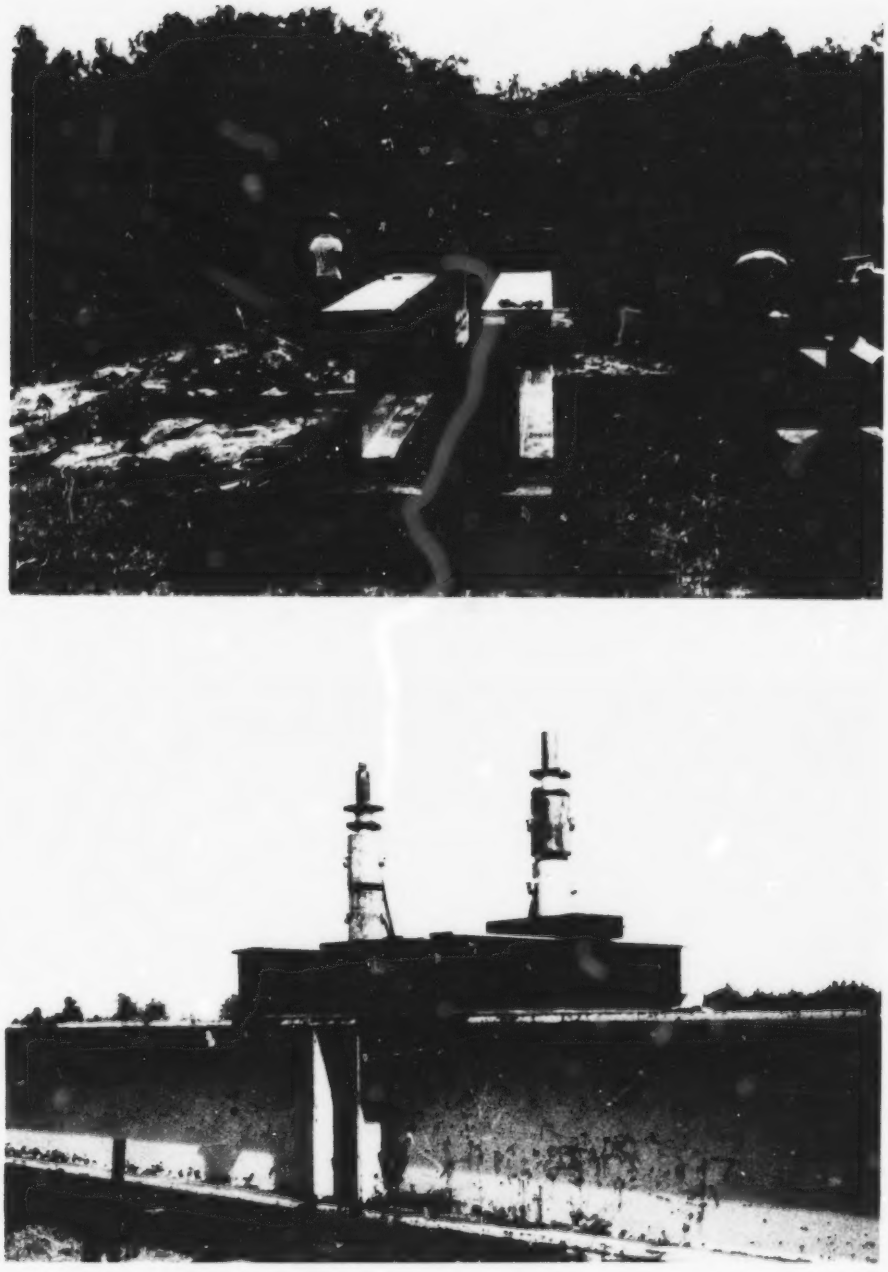

FIGURE 6 - Jacking Apparatus

FIGURE 7 - Dial Gauge and LVDT Installed on Batter Board to Obtain Deflection Readings on the Footings and Tell-Tales

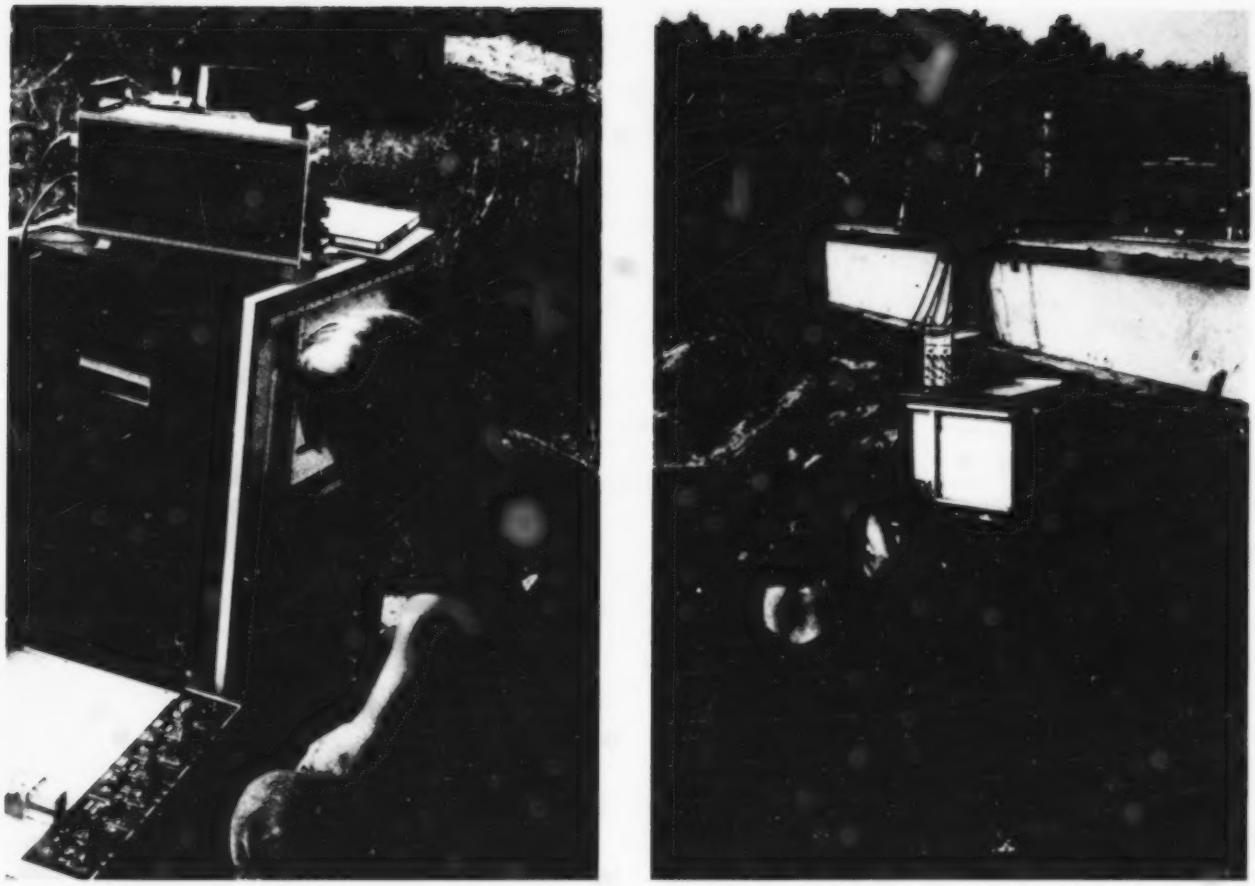

FIGURE 8 - X-Y Plotters were used to Draw Load-Deflection Curves During the Loading of the Footing

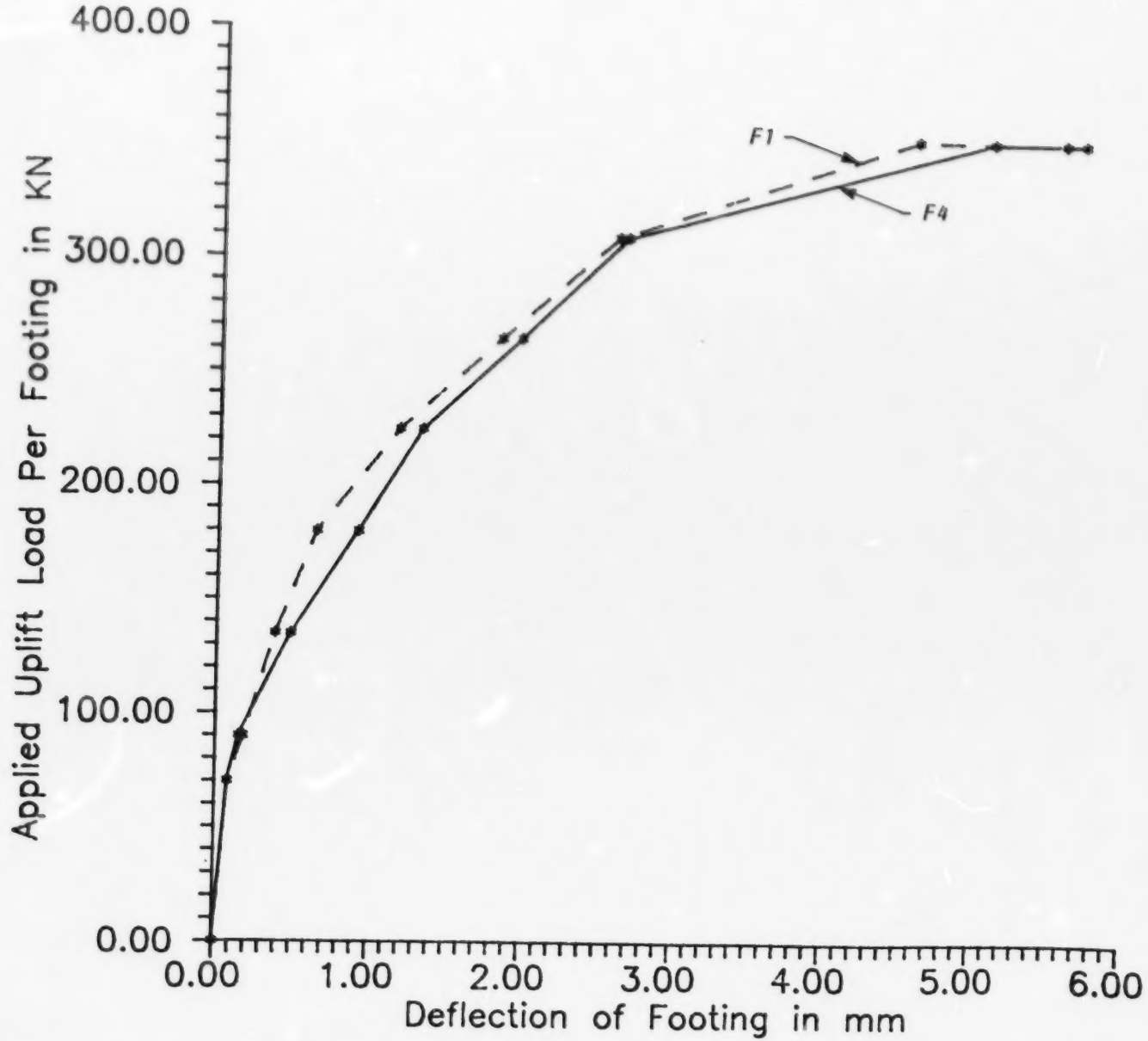

FIGURE 9

APPLIED UPLIFT LOAD VS FOOTING DEFLECTION FOR FOOTING PAIR 1-4

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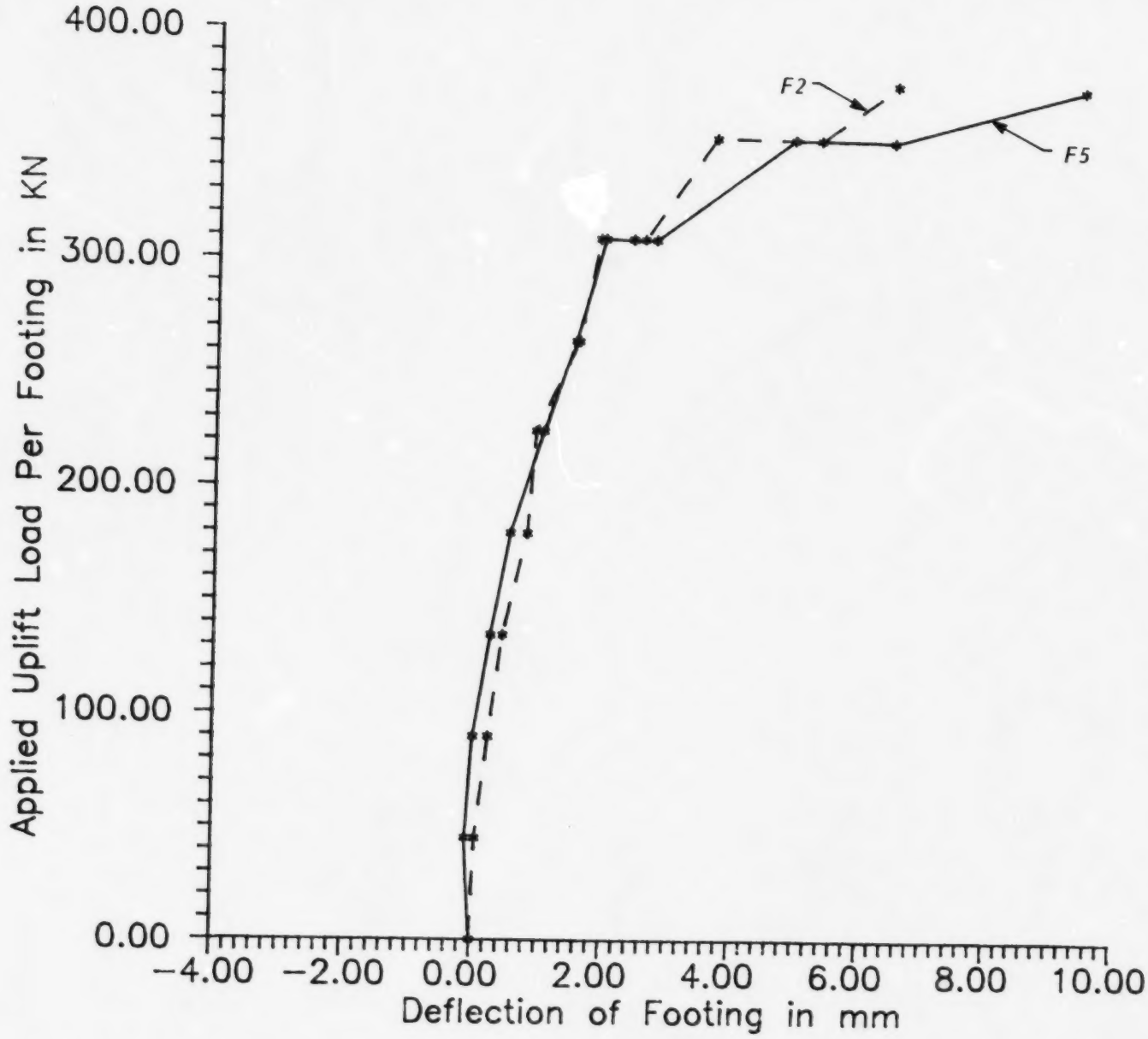

FIGURE 10

APPLIED UPLIFT LOAD VS FOOTING DEFLECTION FOR FOOTING PAIR 2-5

172374-RD

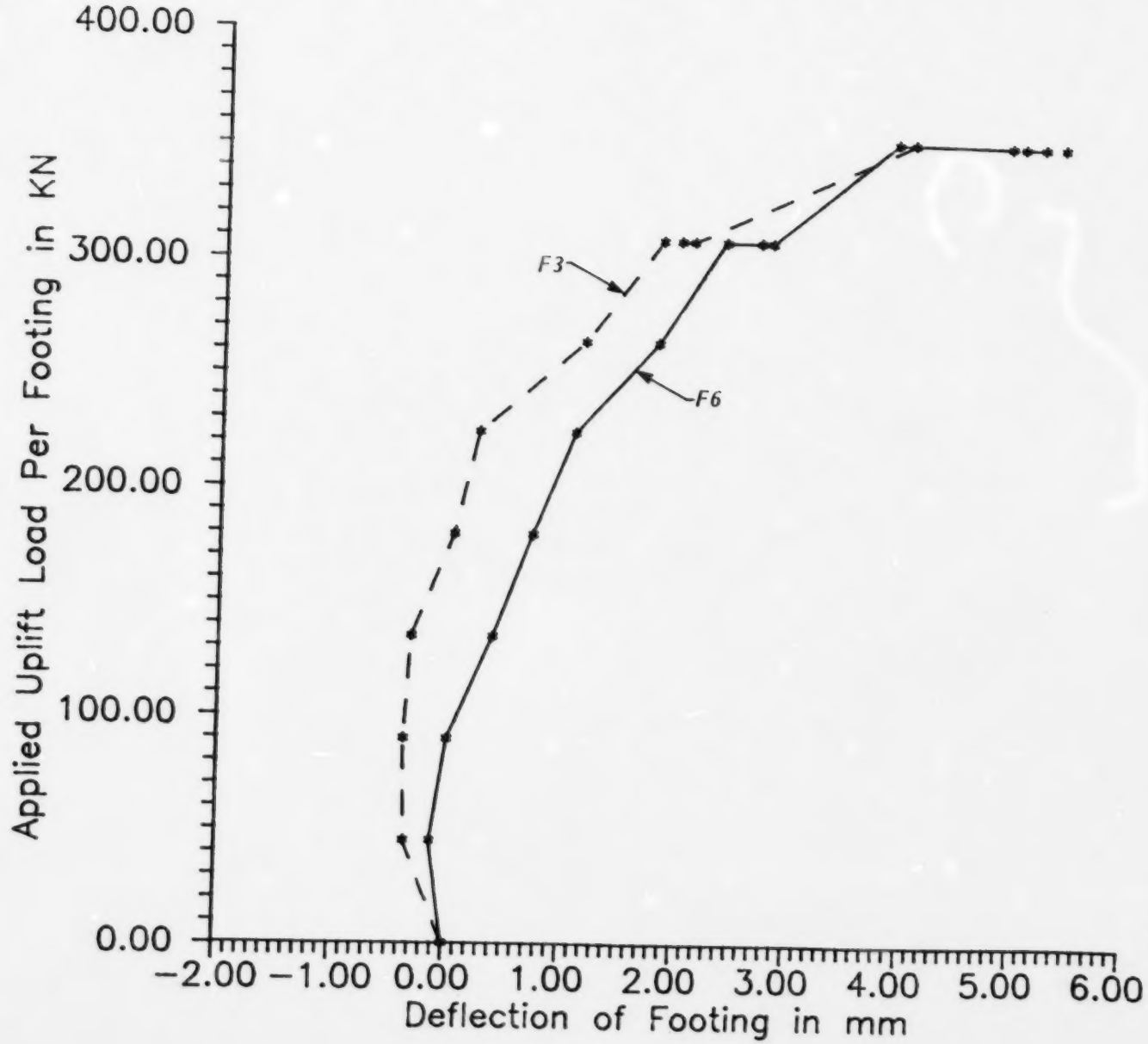

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FIGURE 11

APPLIED UPLIFT LOAD VS FOOTING DEFLECTION FOR FOOTING PAIR 3-6

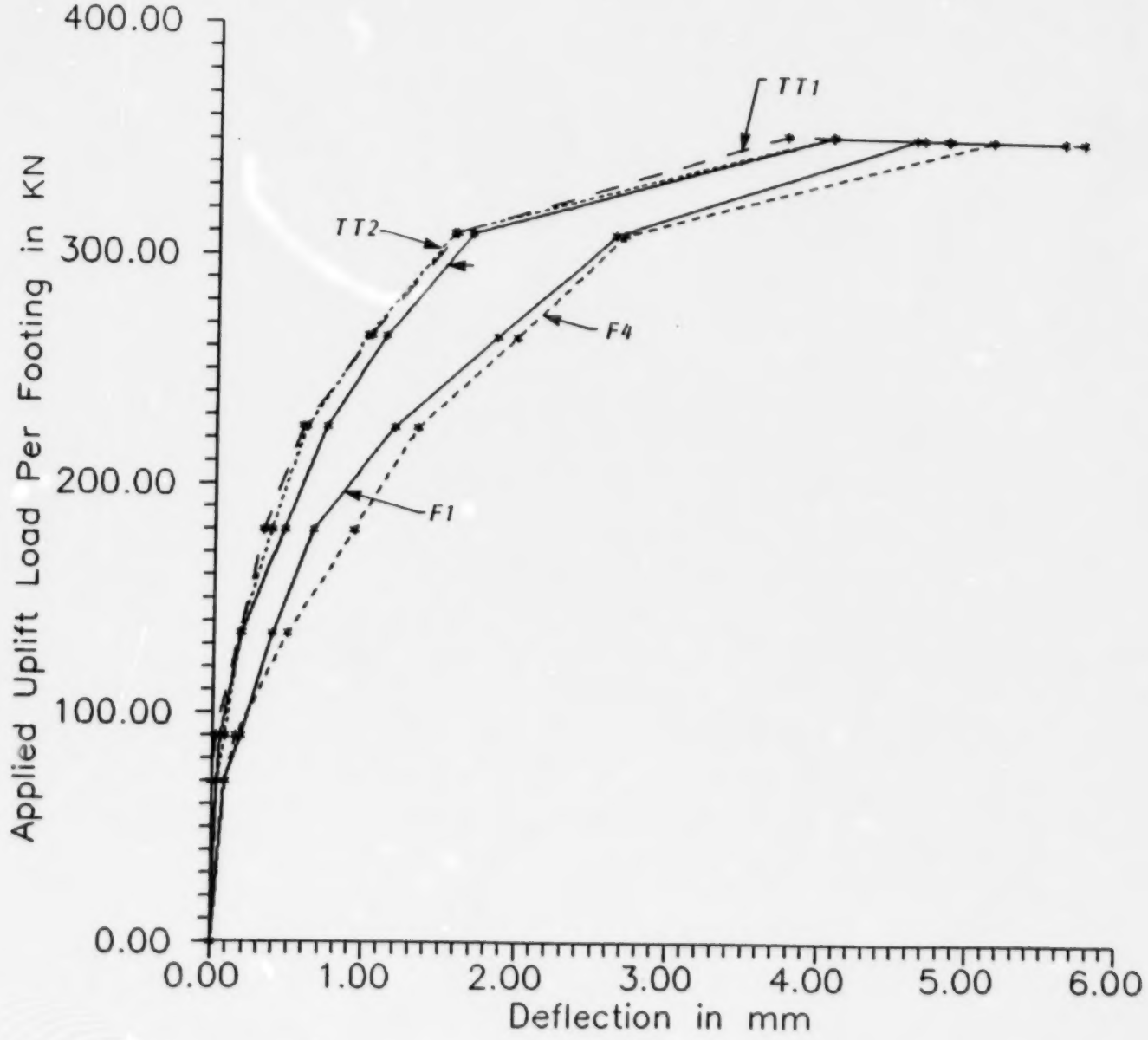

FIGURE 12

DEFLECTION OF TELL-TALES AT FOOTING PAIR 1-4 VS APPLIED UPLIFT LOAD

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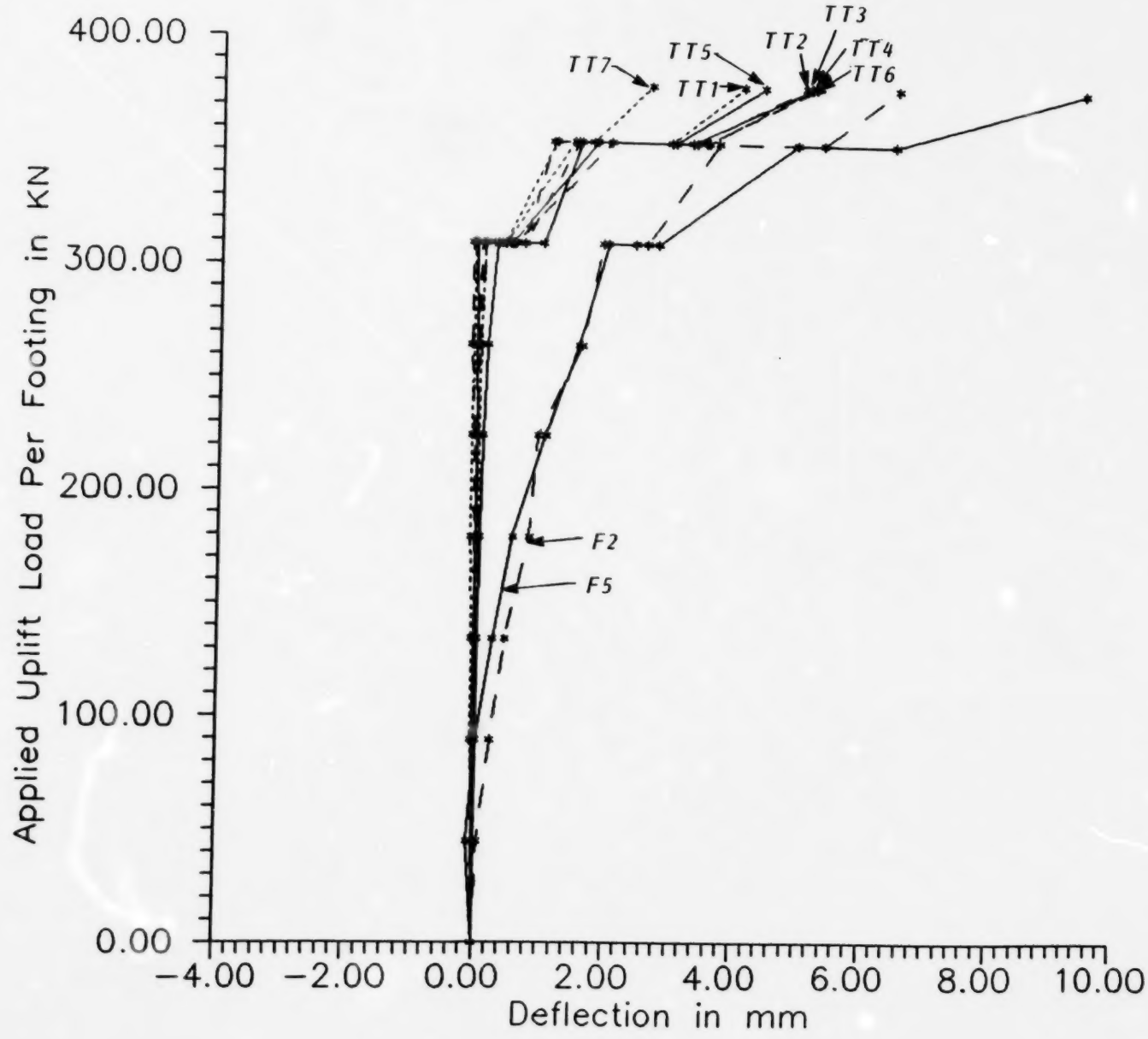

FIGURE 13

DEFLECTION OF TELL-TALES AND FOOTING PAIR 2-5 VS APPLIED UPLIFT AND LOAD

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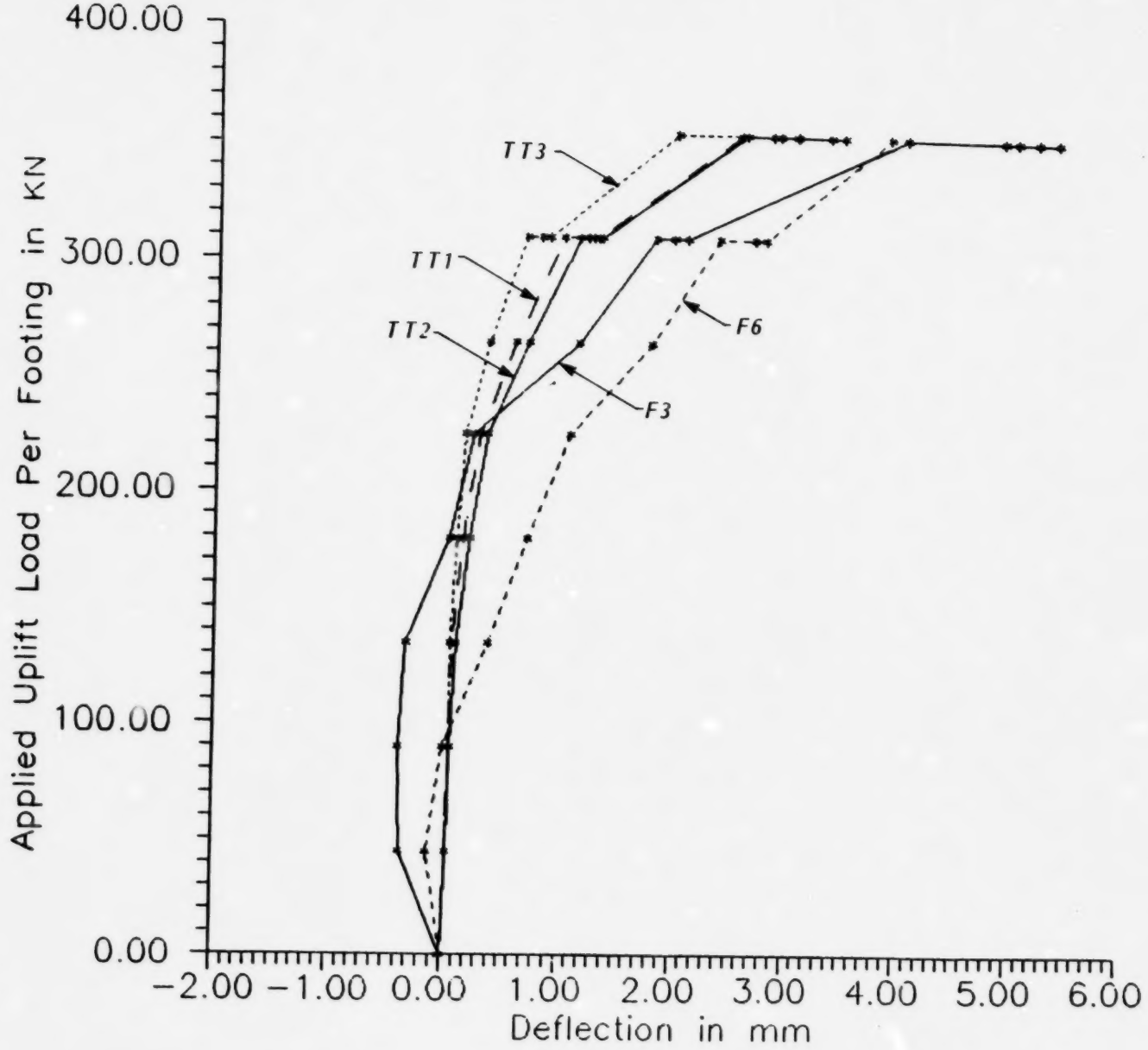

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FIGURE 14

DEFLECTION OF TELL-TALES AND FOOTING PAIR 3-6 VS APPLIED UPLIFT LOAD

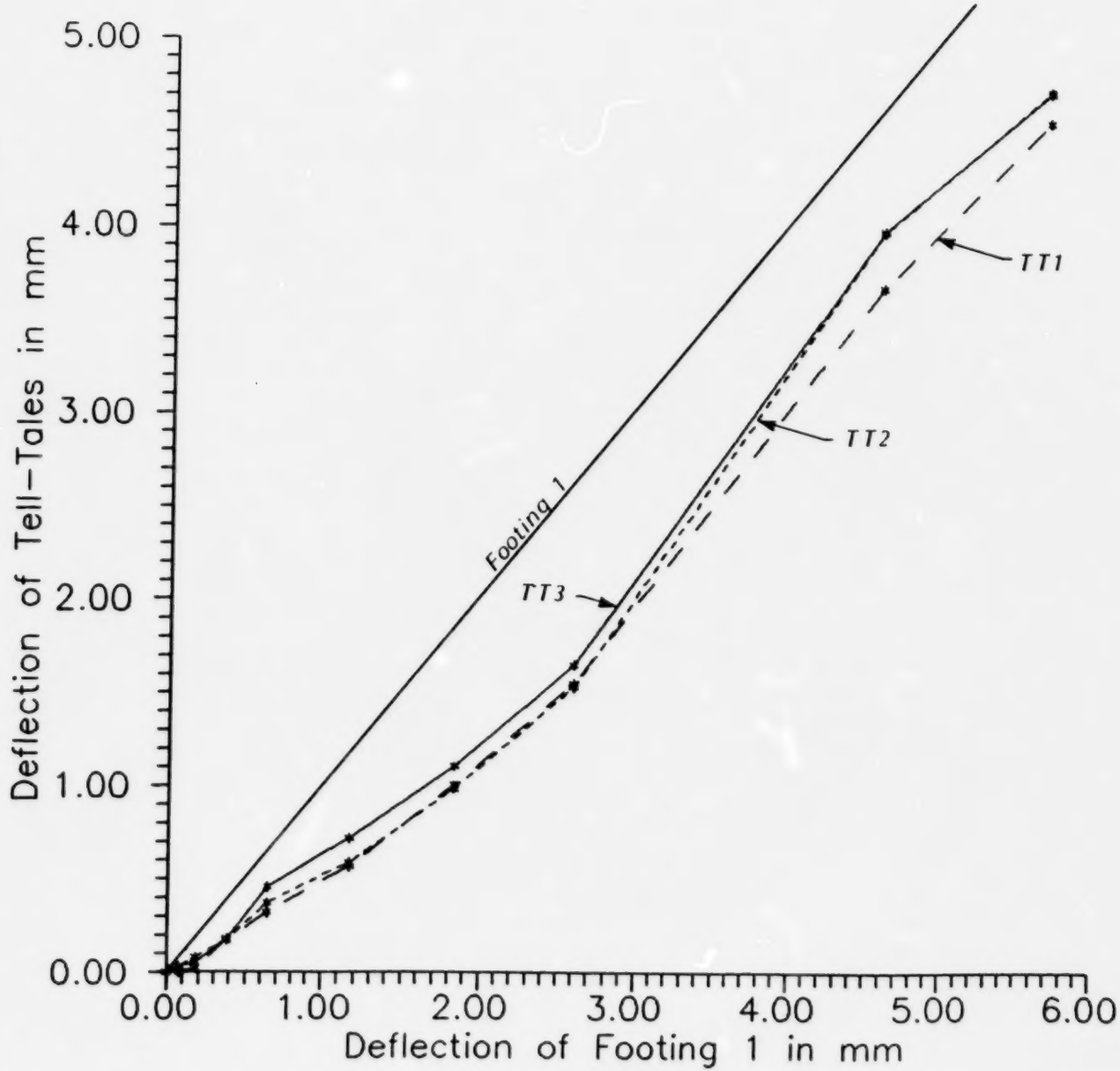

FIGURE 15

DEFLECTION OF TELL-TALES VS DEFLECTION OF FOOTING #1

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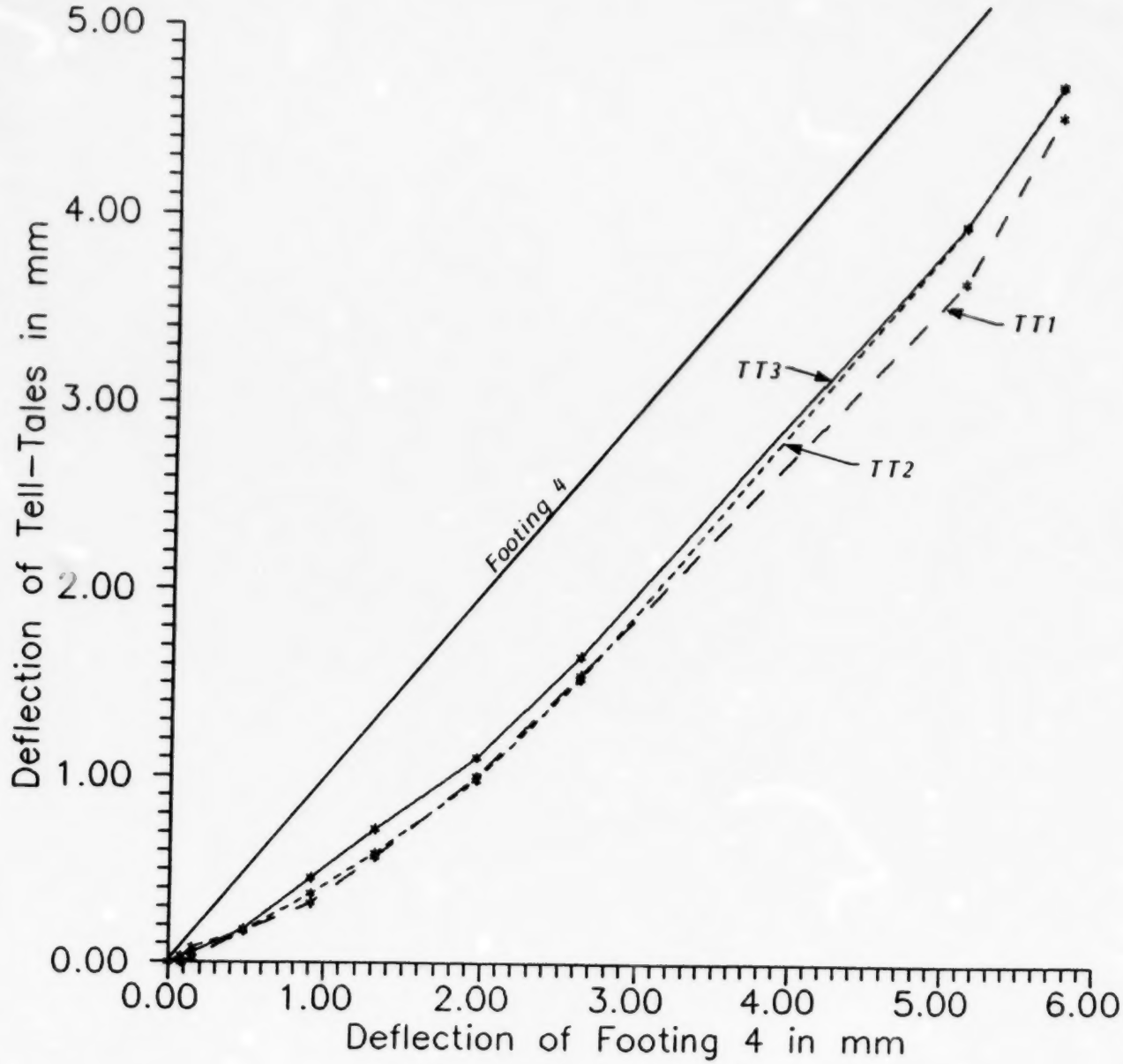
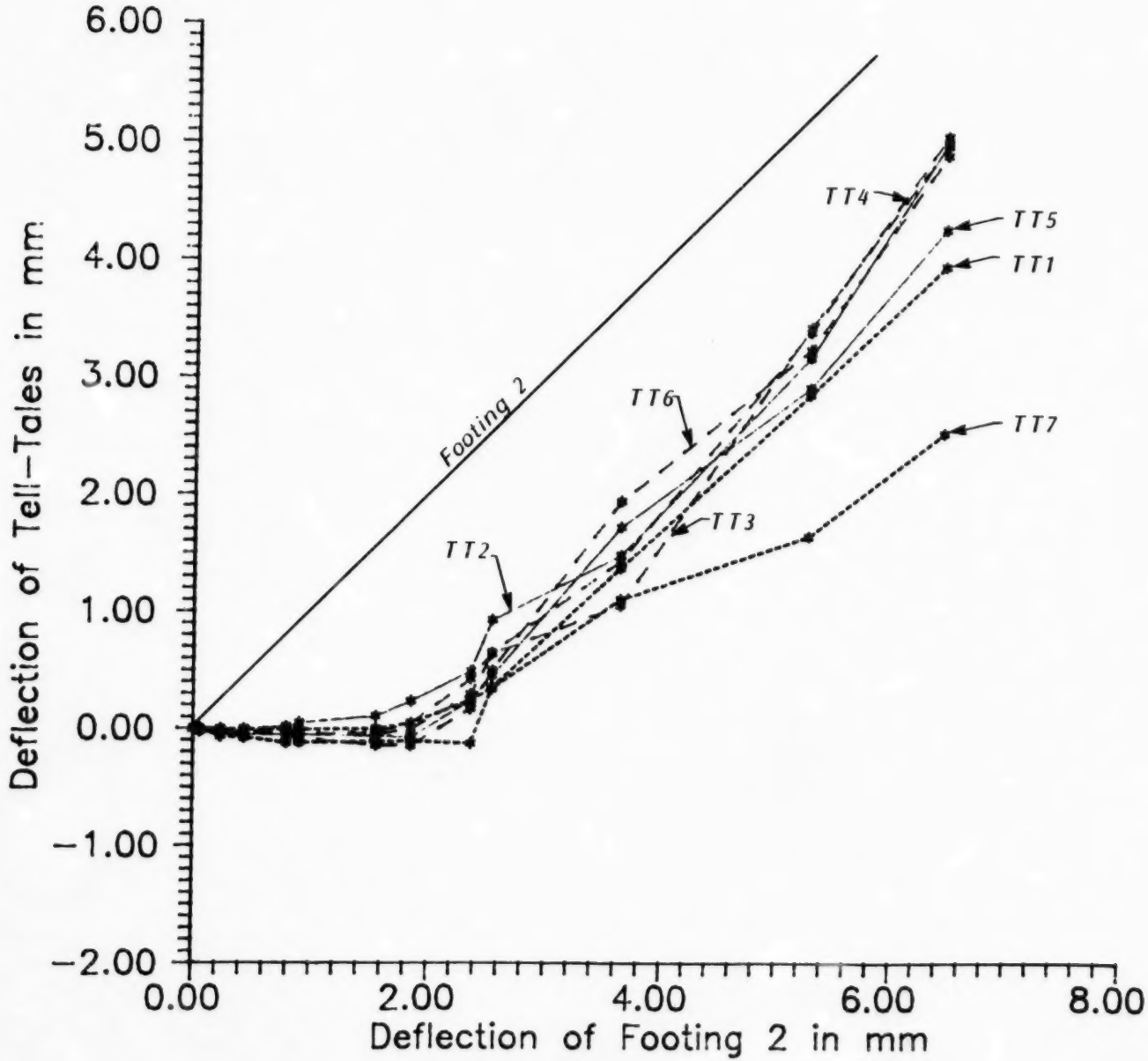


FIGURE 16

DEFLECTION OF TELL-TALES VS DEFLECTION OF FOOTING #4

172380-RD



172381-RD

FIGURE 17

DEFLECTION OF TELL-TALES VS DEFLECTION OF FOOTIN #2

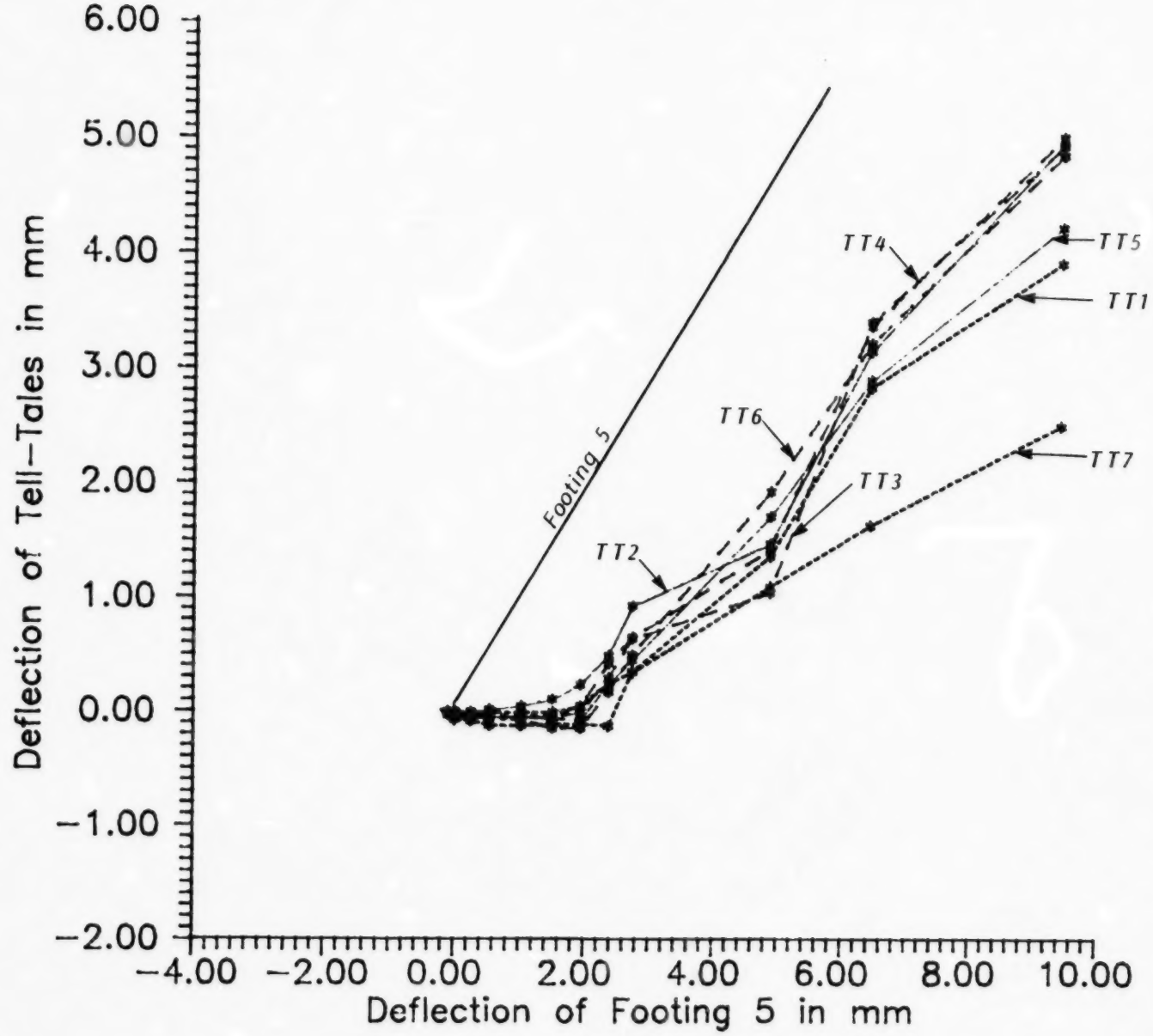


FIGURE 18

DEFLECTION OF TELL-TALES VS DEFLECTION OF FOOTING #5

172382-RD

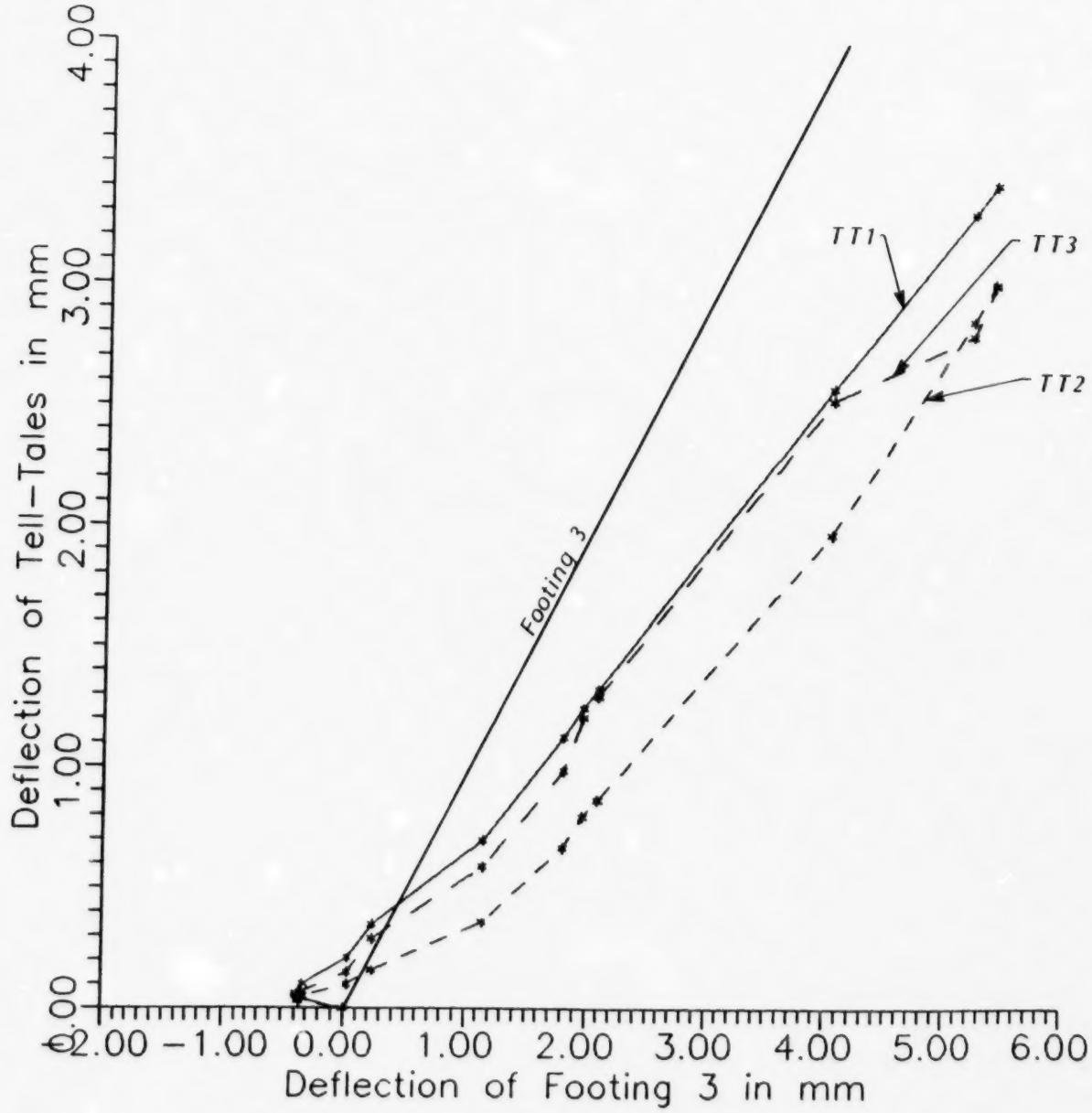


FIGURE 19

DEFLECTION OF TELL-TALES VS DEFLECTION OF FOOTING #3

172383-RD

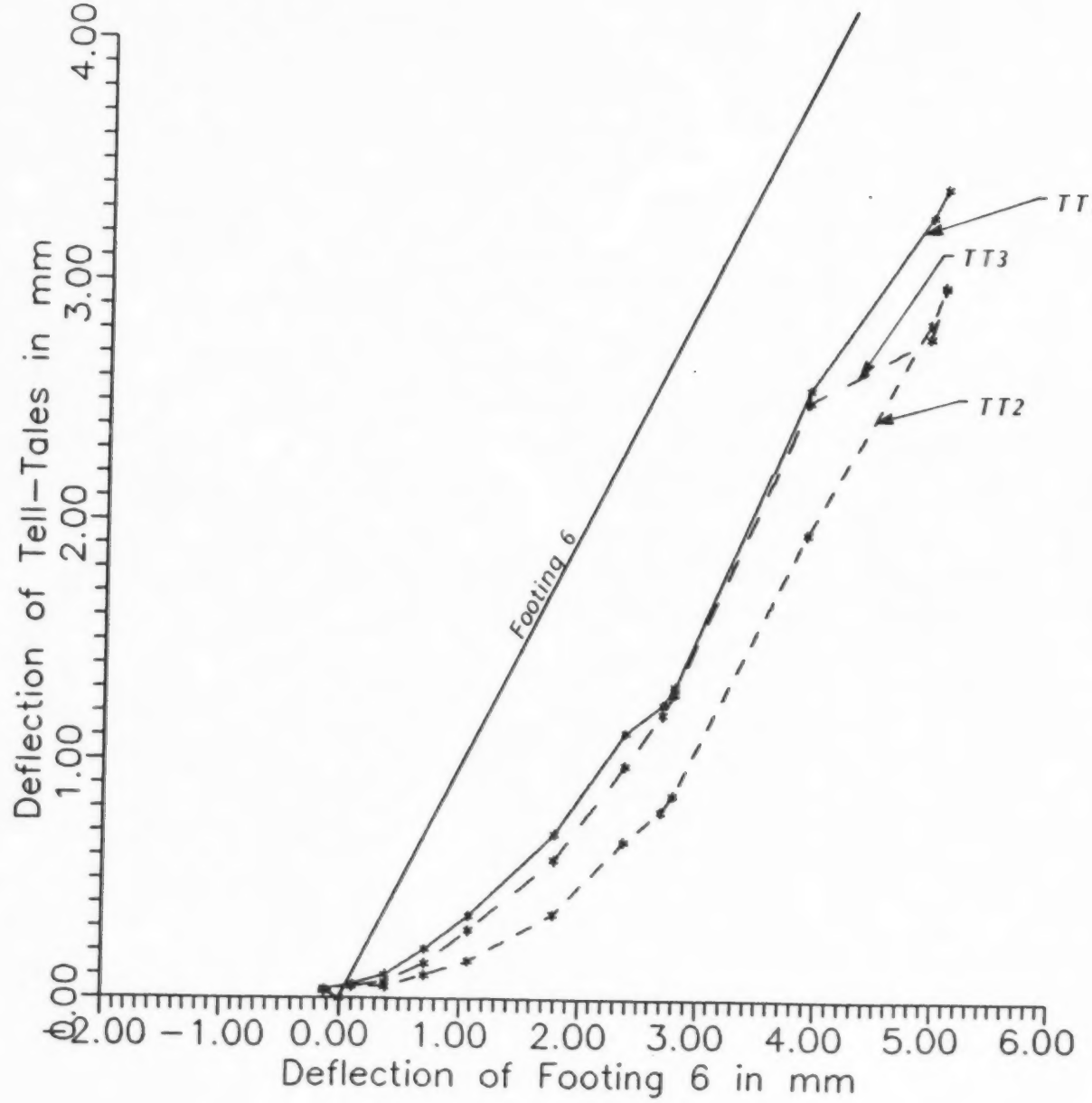


FIGURE 20

DEFLECTION OF TELL-TALES VS DEFLECTION OF FOOTING #6

172384-RD

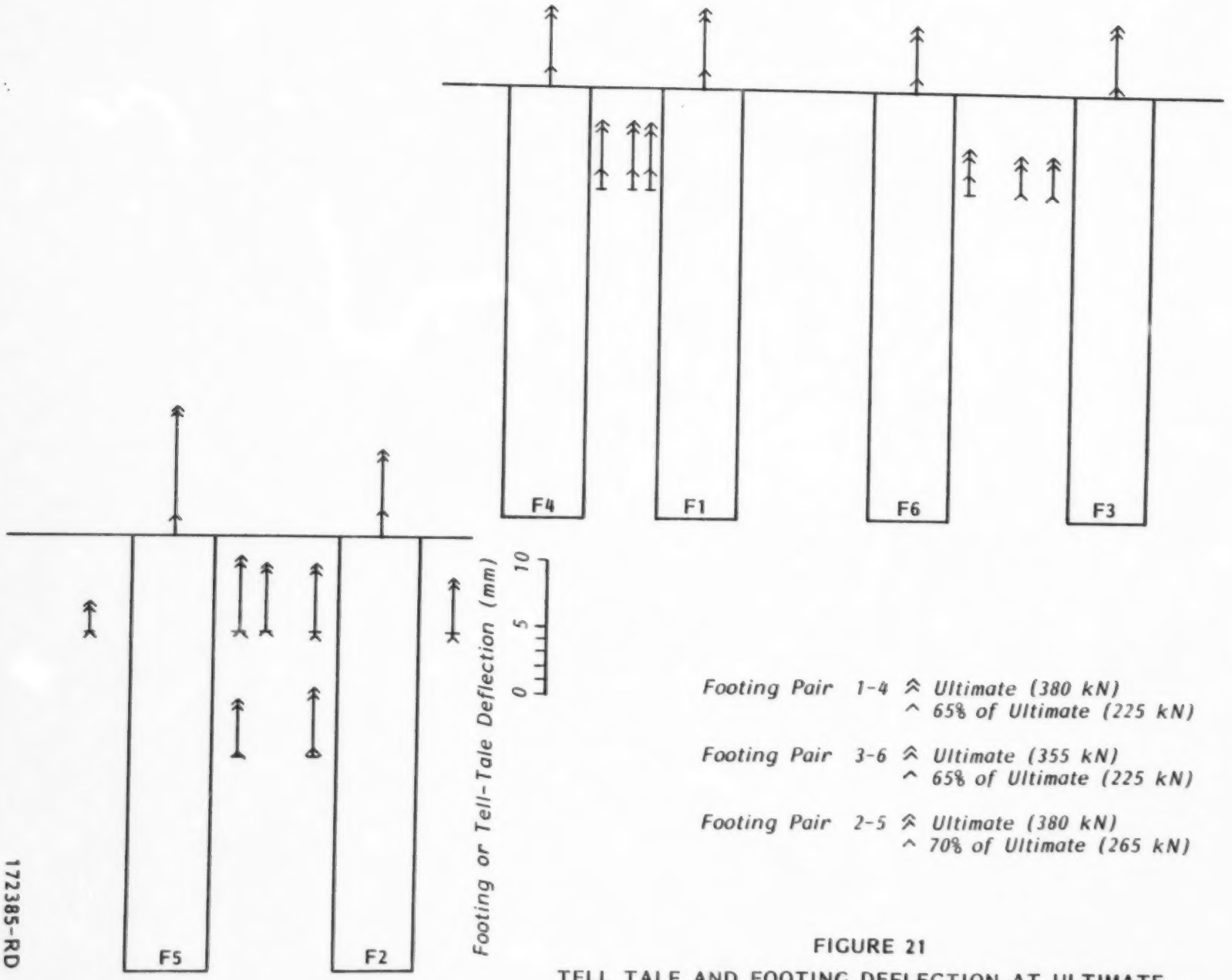
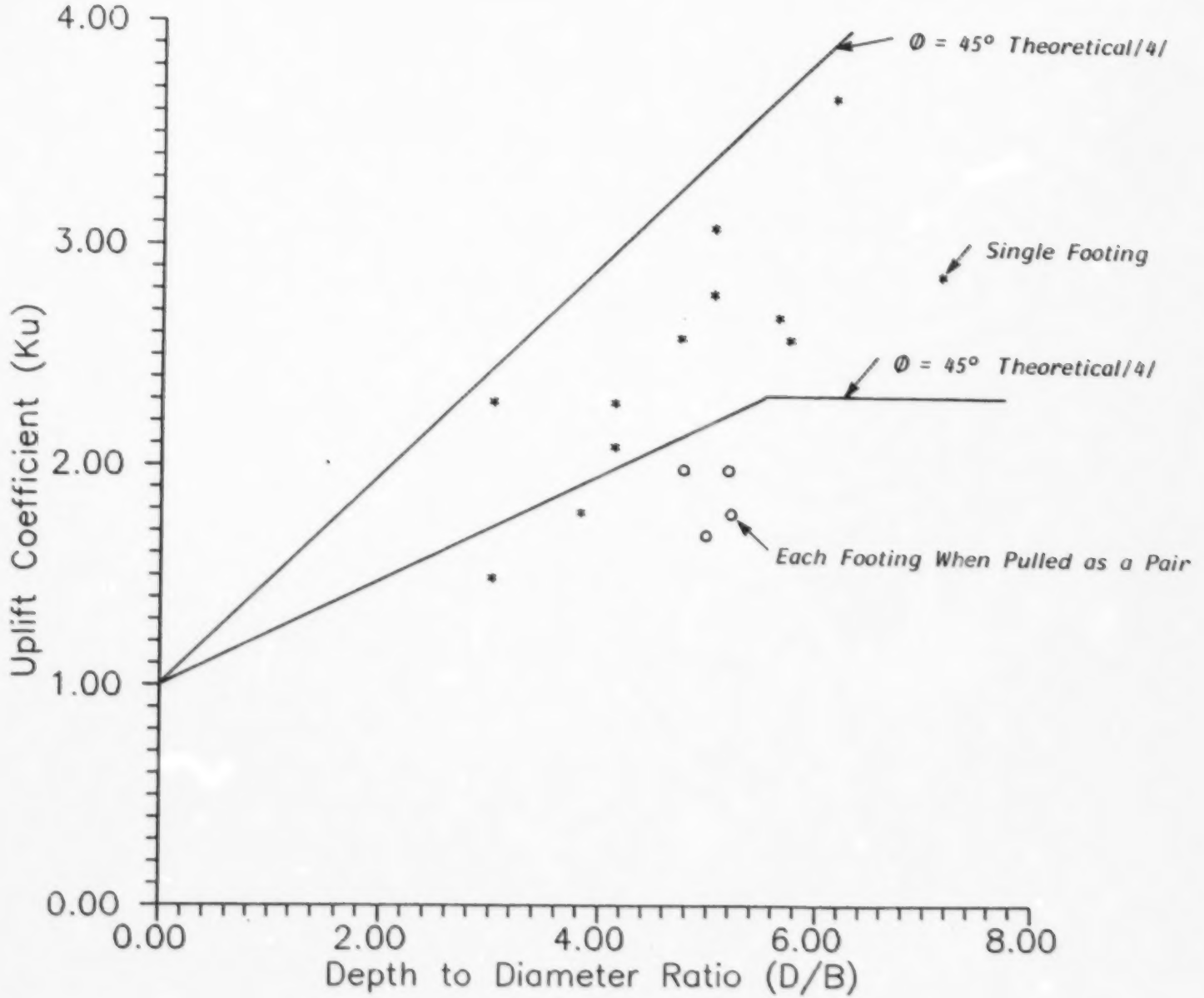


FIGURE 21

TELL TALE AND FOOTING DEFLECTION AT ULTIMATE AND 65-70% OF ULTIMATE



172386-RD

FIGURE 22

UPLIFT COEFFICIENT VS DEPTH TO DIAMETER RATIO FOR FOOTINGS PULLED SINGULARLY AND AS A FOOTING PAIR

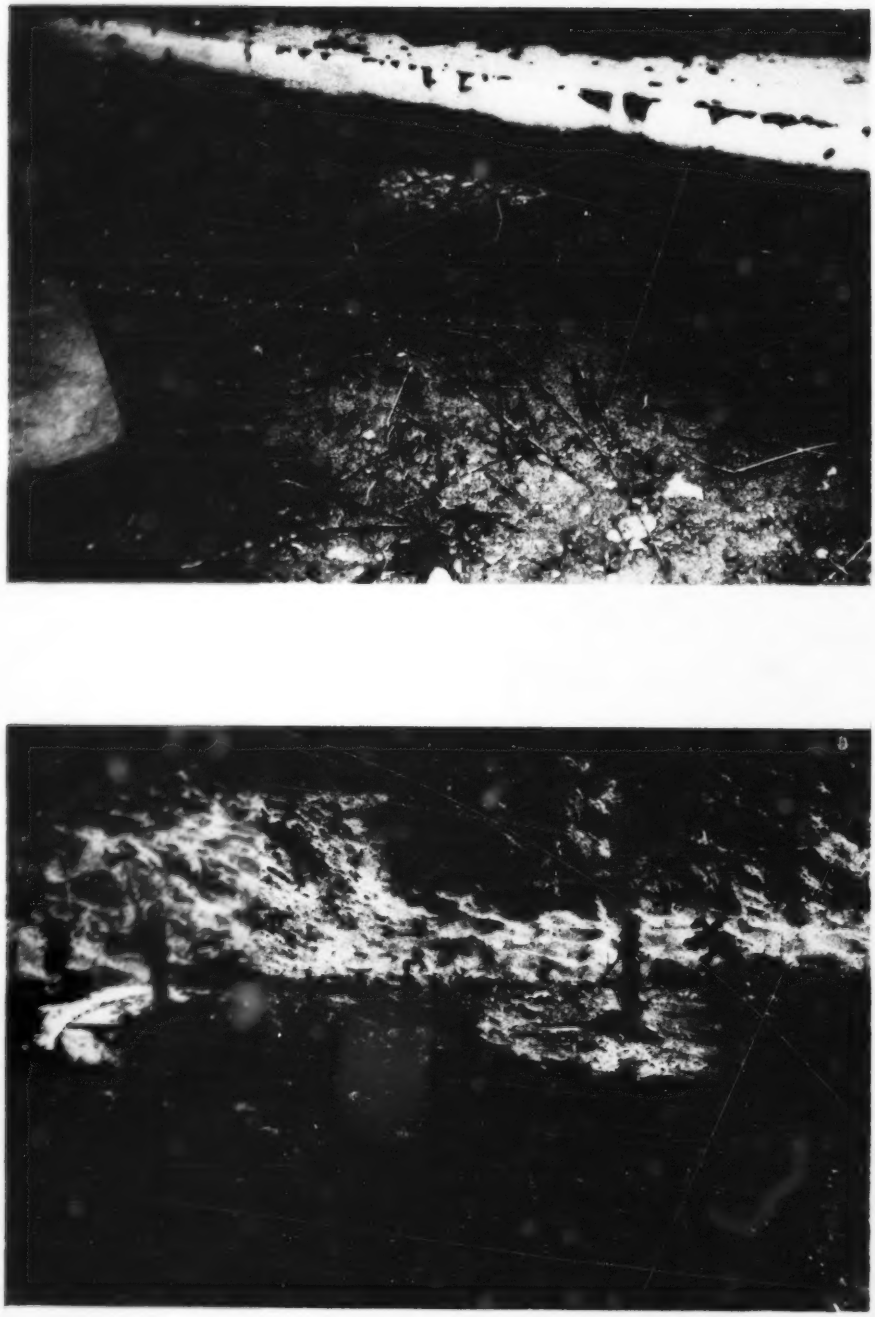
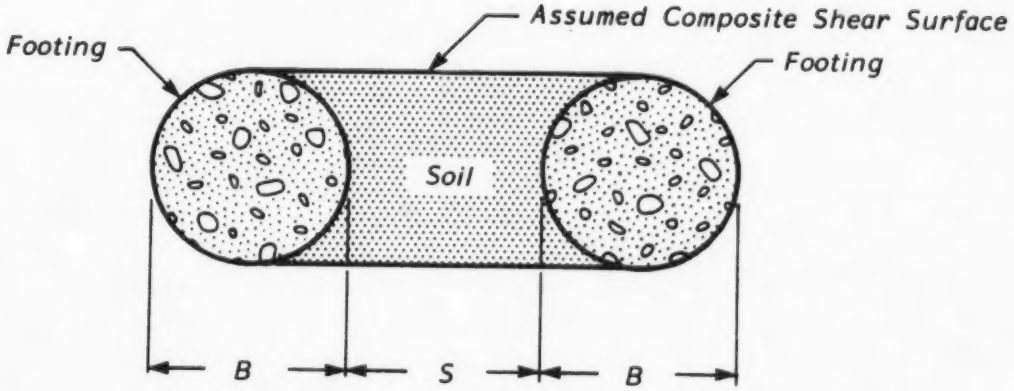


FIGURE 23 - Surface Soil Heave at Failure



$$Q_u = (\pi B + 2B + 2S)\frac{D^2}{2}\gamma K_u \tan\phi + 2W$$

$$K_u = \frac{2(Q_u - 2W)}{(B(\pi+2) + 2S)D^2\,\gamma \tan\phi}$$

FIGURE 24

CALCULATION OF K_u VALUE ASSUMING A COMPOSITE SHEAR SURFACE FOR CLOSELY SPACED FOOTINGS

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